A Report Prepared For:

Nolte Associates, Inc
1750 Creekside Oaks Drive, Suite 200
Sacramento, California 95833

TASK 3b4.6
UNDERPINNING REPORT
RENO RAILROAD CORRIDOR
RENO, NEVADA

Kleinfelder Project Number: 30-1307-10.001

Prepared By:

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Mark Doehring, P.E.
Project Engineer

____
Christine M. Welch, P.E.
Vice President, Regional Manager

KLEINFELDER, INC.
4875 Longley Lane, Suite 100
Reno, Nevada 89502

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I. Introduction

This report provides a discussion of the existing foundations including estimated bearing capacities, concrete slab-on-grade modulus of subgrade reaction and appropriate methods for underpinning the Southern Pacific Railroad Passenger Depot located on Commercial Row between Center and Lake Streets. The building is located within 8 feet of the inside face of the proposed Reno Rail Corridor trench wall. The proposed trench section at the passenger depot will be approximately 30 feet deep, including 23-1/2 feet of open excavation, 7 inch high rails, 10 inch high railroad ties, 18 inches of rock ballast, and a 3-1/2 foot thick reinforced concrete structural slab. The structural slab will be tentatively underlain by approximately 7 feet of jet grouted material. The project will include the construction of an addition to the western end of the depot to allow passenger assess to lower train trench level. It is assumed that the new addition to the passenger depot will be confined within the geometry of the slurry trench wall.

II. Existing Building Foundations

A copy of the foundation plan and foundation details for the Southern Pacific Company Passenger Depot, Drawing 1417, dated February 24, 1925 is attached. The existing building is supported on a series of isolated and continuous conventional shallow foundations. Foundation embedment depths are 39-1/2 to 24 inches for exterior and interior foundations, respectively. There is coal storage/boiler room basement area (approximately 39 feet by 13-1/2 feet by 9 feet deep) located west of the main waiting room along the south side of the building.

Estimated allowable bearing pressures for the passenger depot foundation sections shown on Drawing 1417 are presented in Table 1 below. Allowable bearing pressures for the foundation sections were calculated using Terzaghi's bearing capacity equation and a factor of safety of 3.

TABLE 1

ALLOWABLE BEARING CAPACITIES
SOUTHERN PACIFIC COMPANY
PASSENGER STATION

Foundation Type and Location	Width	Length	Embedment Depth	Calculated Allowable Bearing Capacity (psf) per Terzaghi’s
Continuous, Section A	15″	26½′	39½″	5,700
Continuous, Section B	25” ″	63’-11½″	39½″	6,300
Continuous, Section D	27″	10’-7″	24″	4,500
Continuous, Sections E	29″	3’-3″	39½″	6,500
Continuous, Section G	25″	35½′	39½″	6,300
Square, Pier G	24″	2′	24″	*3,800
Square, Pier S	36″	3′	24″	4,250
Rectangular, Pier T	30″	4½′	24″	4,700
Square, Pier U	30″	2½′	24″	4,000

*Note: A substantially higher allowable bearing capacity was calculated using Meyerhof’s method.

The bearing pressures were calculated based on the subsurface conditions encountered in boring B22 from our geotechnical investigation report referenced in Section VI. Subsurface conditions consist of layered Quaternary Tahoe glacial outwash materials comprised of coarse sand and gravel with frequent cobbles and occasional boulders and a clay matrix within several of near surface layers. Boring B22 is located approximately 30 feet south of the south of the building.

The allowable bearing capacities appear to be substantially higher than what is required for normal long term loading conditions for a single-story structure of wood frame and stucco on brick construction. So without specific subsurface information from within the passenger station building footprint, we recommend limiting allowable bearing capacities to 4,000 psf and 5,500 psf for foundations with an embedment depths of 24 and 39-1/2 inches, respectively.

III. Differential Settlement

Differential settlement beneath the passenger depot depends on the lateral deflection of the trench wall system. Lateral deflection of the wall is dependent on numerous factors, including wall stiffness, depths of staged excavation, the type of bracing, construction workmanship, etc. Several studies have been attempted to correlate past experience with generalized analyses of wall and bracing systems. Clough and O'Rourke (1990) prepared a study in which movement observations for several excavations were tabulated and compared against the stiffness of the excavation support wall systems and the factor of safety against basal heave (Dusenberry, et. al. (1998)). See attached Figure 2, by Clough and O'Rourke. This semi-empirical method was used to estimate the preliminary lateral wall deflections provided below. The estimates are based on a 1m thick slurry wall with an averaged system stiffness of 500 and factors of safety of 2 and 3 against basal heave.

The differential settlement of the soils adjacent to the new trench wall can be roughly estimated by assuming the ratio of maximum vertical soil movement at the wall face to lateral wall movement be set to 1. Attached is Figure 3, by Clough and O'Rourke, which provides recommended settlement profiles for various soil conditions. We used settlement profile a) sands for our preliminary foundation settlement estimates for the passenger depot.

TABLE 2

EXTERNAL WALL FOUNDATION SETTLEMENTS FOR THE PASSENGER DEPOT
BASED ON ESTIMATED FACTOR OF SAFETY AGAINST BASAL HEAVE

Estimated Factor of Safety against Basal Heave	Wall Support System	Estimated Lateral Deflection	Estimated Settlement at the Passenger Depot North Wall Foundation	Estimated Settlement at the Passenger Depot South Wall Foundation
2.0	Tieback (3.5m vertical spacing)	0.76″	0.71″	0.27″
3.0	Tieback (3.5m vertical spacing)	1.05″	0.99″	0.37″
2.0	Cantilevered	1.41″	1.33″	0.49″
3.0	Cantilevered	3.10″	2.91″	1.09″

The semi-empirical method developed by Clough and O'Rourke does not take into account all the individual components that contribute to lateral wall deflection, such as poor workmanship and as such may underestimate lateral wall deflections. However, the wall deflections are with in the range of 0.2% to 0.4% of the wall height, which is typical for a well-constructed slurry trench wall with an excavation support system (tiebacks or bracing). On many large projects, the issue of lateral wall deflection is usually handled by specifying a maximum acceptable lateral wall deflection, typically 1 inch for urban areas, within the project documents. Note: There is usually less ground loss or lateral wall movement associated with a tieback supported cut compared to a braced cut. In many instances, this is due to the ability to place anchors once the excavation level reaches the intended anchor level, rather than the necessity of having to excavated beneath the braced level in order to place struts at the required elevation.

The wall deflection estimates are within the range of the maximum deflections specified in the technical provisions for the Alemeda Corridor, Mid Corridor Design-Build Project. The Alemeda Corridor is a depressed train corridor with similar geometry currently under construction in Los Angeles, California. The Alemeda Corridor, Mid Corridor Design-Build Project specification limits the lateral deflections as follows:

a) Lateral deflection at top of wall (walls laterally supported at the top)
   - Dead Load + Soil Pressure 19 mm (3/4 inches)
   - Dead Load + Soil Pressure + Live Load Surcharge 38 mm (1-1/2 inches)

b) Lateral deflection at top of wall (cantilever retaining walls)
   - Dead Load + Soil Pressure 38 mm (1-1/2 inches)
   - Dead Load + Soil Pressure + Live Load Surcharge 76 mm (3 inches)

A copy of part of Section 3-18 of the Alameda Corridor specifications is attached.

IV. Modulus of Subgrade Reaction for Concrete Slab-On-Grade Floors

Typically, a modulus of subgrade reaction, k, is used in the design of concrete slab-on-grade floors. The modulus of subgrade reaction is the ratio between the pressure on the surface area, q, and the resulting deflection, y.

$$k = q/y$$

For concrete slab-on-grade floors constructed in the downtown Reno area, we recommend for preliminary design using a modulus of subgrade reaction of 300 pci/in. The value of subgrade reaction is dependent on the size of the loaded area and the type of loading. Our recommendation was based on the following assumptions:

a) Subgrade consists of a minimum of 6 inches of compacted native granular subgrade;
b) A floor slab length to width ratio of approximately 1.5; and
c) Short term (live load) loading conditions.

Settlement beneath the floor slab can be assumed to be decreasing linearly from north to south between the building external wall foundations. Estimated settlements between the north and south exterior foundations at the passenger depot are provided above in Table 1. Attached is Figure 3, Case a) by Clough and O'Rourke, which provides the recommended settlement profile for beneath the existing building floor slab.

V. Underpinning of Existing Foundations

The need for underpinning the passenger depot foundations and floor slab will depend largely on how much ground loss occurs beneath the building due to the trench wall deflecting laterally and how much differential movement the existing structure can withstand. Provided underpinning is necessary to arrest and prevent structural movement, we anticipated micro-piles would be the most effective way to underpin the structure.

Micro-piles are small-diameter (seldom greater than 6 inches in diameter), bored, grouted in-place piles constructed with some form of steel reinforcement such as open ended steel pipe or vertically placed reinforcing bar. These piles can be designed to sustain axial and/or lateral loads. Micro-piles can be installed in limited access areas and placed through and bonded with the existing structure without the need for a pile cap. Micro-pile installations typically cause less noise and vibration than conventional piling techniques and should not cause damage to adjacent structures or affect nearby ground conditions. Due to the coarse nature on the underlying soil conditions at the passenger depot, we anticipate that a rotary percussive drill system will be required to install the underpinning system.

The basic philosophy of micro-pile design differs little from that required for other pile types. The design of micro-piles involves the following steps:

a) Structural design of the steel micro-pile: Typically, micro-piles are designed not to exceed 80% of the minimum yield stress (f_y) of the steel reinforcement. When underpinning lightly loaded structures, normally reinforcement is provided in the form of a single reinforcing bar grouted into the center of the micro-pile boring.

b) Design of grout to steel bond: For design purposes, the grout to steel bond can be estimated to be on the order of 300 psi for deformed steel and 200 psi for smooth bars and pipe, assuming the grout mix consists of neat cement with an average minimum 28-day compressive strength of 5,000 psf.

c) Determination of the ground to grout bond: The ground to grout bond depends largely on the method of pile grouting. Methods for pile grouting include gravity head placement only, pressure grouted through the casing during casing withdrawal, primary grout placed under gravity head, then one phase secondary "global" pressure grouting and primarily grout placed under gravity head, then one or more phases of secondary "global" pressure grouting. For preliminary design assuming gravity flow grout only, we recommend using an ultimate grout to ground bond of 3,000 psf. The factor of safety of 2.5 is recommended for the grout to ground bond for typical underpinning design. Verification of the grout to ground nominal bond strength assumed in design via pile load testing is essential to ensure structure safety.

The <u>load bearing section</u> of the micro pile should extend below the Rankin failure plane. This can roughly be estimated to be a plane extending behind the trench wall from the top elevation of the reinforced concrete slab at an angle of 64 degrees from the horizontal to the finished ground surface. For micro-piles installed beneath the passenger depot northern external foundations, the load bearing section of the underpinning system would start at a depth of approximately 22 feet below the existing ground level. The southern external foundations for the passenger depot would be outside the Rankin failure plane. The bearing length of the pile will depend load of the structure and pile spacing. Spaces for micro-piles for underpinning systems are typically on the order of 3 to 6 feet on center and depend on the amount of steel reinforcement within the foundation section. No individual pile capacity reduction for group considerations is necessary for piles with center to center spacing greater than 3 times the pile diameter.

VI. References

Draft Memorandum, *Geotechnical Input for Amtrak Passenger Depot, American Railroad Express Station (Men's Club) and Freight House, Reno Rail Corridor*, Kleinfelder, Inc., September 28, 2000.

Memo, *Reno Rail Corridor Geotechnical Questions, File 80610001*, Stantec, September 6, 2000.

Geotechnical Engineer Report, Reno Railroad Corridor Draft EIS, Reno, Nevada, Kleinfelder, Inc., May 9, 2000.

Draft, *Micropile Design and Construction Guidelines Implementation Manual* (May 1999), US Department of Transportation, Federal Highway Administration.

Dunsenberry, D. O. and J. R. Davie (1998), *Effects of Construction on Structures*, Geotechnical Special Publication Number 84, ASCE, Geo Congress '98.

Foundation Details, Southern Pacific Co. Passenger Station, Reno – Nevada, Drawing 1417, February 24, 1925.